

Incremental Launching Challenges on Mount Henry Bridge

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SYNOPSIS

As part of the new Southern Suburbs Railway construction in Perth, a new Mount Henry Bridge was constructed alongside and partially overhanging the existing bridge to widen the freeway. The new bridge was incrementally launched which provided the design and construction team with a number of unique challenges, particularly in maintaining clearance to the old structure. The bridge spans were halved for launching using temporary piers which significantly reduced deflections enabling tighter control over the launch. However the temporary piers provided their own unique challenges and had to be propped to the existing bridge and adjacent permanent piers to improve the buckling behaviour of the slender pile sections, as well as preset to reduce vertical loading. Testing and monitoring of the insitu pier movements was used to confirm the design assumptions, in particular the lateral restraint provided by the soft river alluvium material. The tight construction schedule required launch cycles to be minimised which was achieved by adopting a two staged pour for the deck and minimising the prestressing operations.

1. INTRODUCTION

Incremental launching is a well established construction technology, especially in Perth. However the new Mount Henry Bridge presented some particular challenges which required fundamental bridge engineering consideration for both design and construction.

The principal difficulty was that temporary piers had to be installed at the middle of the 76m spans, which were considered too long for launching without intermediate support. Because the tight construction schedule and compatibility with other piling operations required the use of 600mm diameter steel piles, the temporary pier design resulted in a group of four piles which were very slender for their length of over thirty metres into very soft mud. The buckling stability of this system relied on the lateral restraint capacity of this soft mud. In addition, the permanent concrete piers were founded on groups of 14 to 18 similar steel piles, resulting in a large discrepancy between the vertical stiffnesses of the temporary and permanent piers.

This paper discusses the analysis of the temporary and permanent piers and their interaction with the superstructure during launching. It also discusses how the soft soil restraint was modelled, and how field tests were devised to “calibrate” it. Descriptions of various construction design problems and solutions arising from these calculations are then given. These include adjustments built into the temporary launch bearings, the lateral and longitudinal temporary pier bracing, and the monitoring carried out during construction, which was fed back into the calculations for construction design.

The tight design and construction schedule set by the contract was an overriding factor which governed a lot of the design decisions. Staging of the deck construction, devised to optimise the speed of construction, was facilitated by the use of temporary piers which themselves had to be designed to fit into the construction schedule.

2. MOUNT HENRY BRIDGE BACKGROUND

The original Mount Henry Bridge, located nearly ten kilometres south of Perth and opened in 1982, was built to carry the Kwinana Freeway across the Canning River. The 660m long, 28.8m wide, segmentally constructed, twin cell concrete box girder was designed to carry three lanes of freeway traffic in each direction with shared paths on the lower cantilevers.

The river crossing at the Mount Henry Bridge site required widening as part of the new Southern Suburbs Railway project to accommodate two lines of passenger rail along the middle of the Freeway in addition to the existing freeway traffic requirements. The main bridgeworks for the rail project, including the Mount Henry Bridge, the Narrows Bridge and the Canning bus bridge amongst others, were grouped under one contract called “Package E”, which was awarded to Leighton Contractors in January 2004. The Leighton Contractors design team for Package E comprised of Coffey Geosciences, GHD and Wyche Consulting. The tender called for a symmetrical widening of the existing Mount Henry Bridge, but the winning Leighton Contractors bid included a completely independent new bridge to be constructed on the western side and fitting into the existing bridge. The northbound carriageway of the freeway would be transferred to the new bridge allowing the existing bridge to carry the southbound carriageway and the two railway lines as shown in Figure 1.

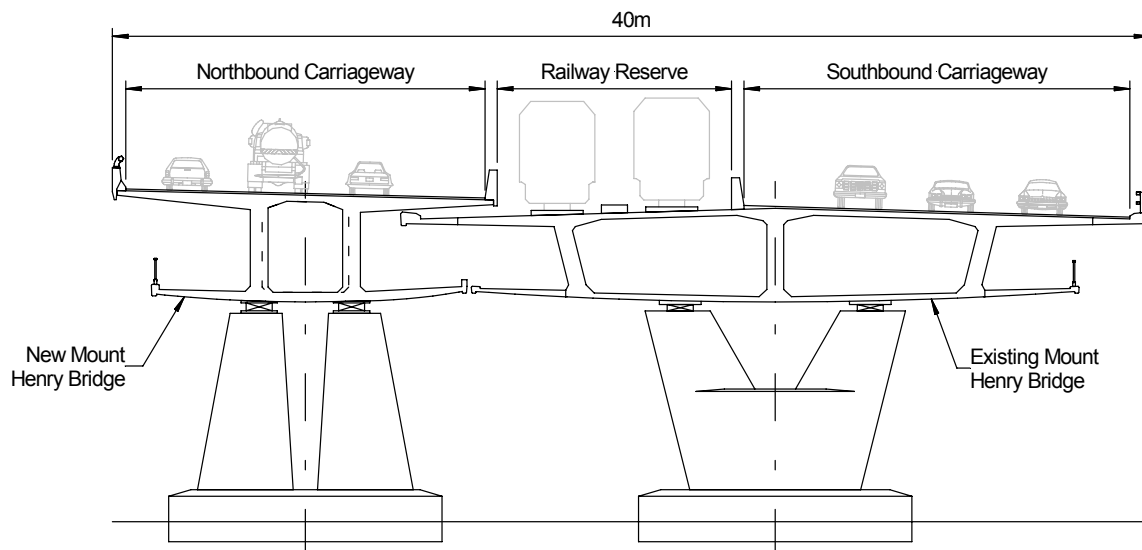


Figure 1: Mount Henry Bridge cross section

Restrictions on land availability, particularly at the northern (Mount Henry) end of the bridge, meant that to achieve the required freeway cross section, the new bridge had to fit into the existing bridge with the new deck overhanging the old. The new deck cantilever would clear the old kerb by a minimum of 85mm and the gap between the

lower cantilevers of the two bridges would be approximately 185mm for the entire length of the bridge. Extensive survey was carried out to ensure these clearances could be achieved for launching the new bridge along a circular curve.

A number of factors influenced the final shape of the cross section, including functional requirements for the deck and a lower level shared path, aesthetic requirements, structural design requirements, and construction requirements. The existing Mount Henry Bridge is considered to be an important Perth landmark because of its location and unique aesthetic qualities which meant the architectural requirement for the new bridge to complement the existing bridge was of utmost importance. The final structural form was heavily influenced by Leighton Contractors' architect, Parry & Rosenthal Architects, in combination with Main Roads' architect.

The Leighton Contractors team had a two year contract to carry out the Package E works which included the design and construction of the new Mount Henry Bridge as well as strengthening of the existing bridge. To minimise impact on the heavily trafficked Freeway, the northbound carriageway had to be moved to the new bridge before the majority of the modification works to the existing bridge could be carried out, hence both lots of work could not be carried out concurrently which put additional pressure on the already tight schedule. From the outset, the realisation was that the timeframe for the project was exceptionally tight and this factor guided a lot of the decision making throughout the job.

3. CONSTRUCTION METHOD

Initially a number of construction methods were put forward for consideration. These included incremental launching, segmental balanced cantilever, and segmental span-by-span. The original Mount Henry Bridge was constructed using a segmental span-by-span approach. However in the case of the new bridge, segmental methods were not preferred because of the difficulty in handling and placing segments to fit around the old bridge. Other reasons for not using segmental construction included: the additional prestress resulting from the requirement for net compression across segment joints; and more importantly, incremental launching was considered to be a quicker construction method for the size of the project.

Once incremental launching was selected as the preferred construction method, the next decision to be made was whether or not to utilise temporary intermediate supports during the launch. Launching unsupported over 76m spans with a 4m deep section was outside the known experience of the team so relatively early it was decided that temporary piers were required. By using temporary intermediate supports to achieve 38m launched span lengths, Leighton Contractors was able to re-use an existing launch nose, albeit with some modification. This not only saved money on the launch nose, but also reduced setup time. The use of temporary supports also gave the construction team tighter control over displacements during launching which was critical in ensuring the new bridge would not contact the old bridge during construction. In addition, the shorter spans required less concentric prestress which had cost and time benefits, as discussed later. For Leighton Contractors in this case, the benefits of adopting the shorter spans were significant.

4. TEMPORARY PIERS

4.1 Piles

A major challenge that faced the team was to design a temporary pier system that was cost effective and would not delay the works with deck construction expected to closely trail the pier construction. The preference was to use similar steel pile sections used for the permanent piers as they could be procured relatively quickly. Two similar thin walled circular hollow section sections were available, the larger of which was a 660mm diameter pipe with a wall thickness of 11.6mm fabricated from steel with a yield strength of 448MPa.

Geotechnical investigation by Coffey (Package E Design Reports) confirmed previous test results and showed there was a layer of weak Swan River Alluvium, up to 25m deep, overlying a strong South Perth Siltstone. The soft alluvium layer was expected to provide little lateral restraint to the piles, which meant the most critical temporary pier piles would be effectively unsupported for almost 30m from the top of the siltstone to the underside of the headstock beams.

Buckling analysis of the pier frame, using lateral soil spring values provided by Coffey, revealed that the alluvium layer did in fact provide some restraint against buckling, however the effective length (L_e) of the piles was in the order of 23m (refer Figure 2). The modified member slenderness (λ_n) of the piles was calculated to be 135, hence the section would be only about 35% efficient for pure compression. Based on this assessment, a minimum of seven piles was required at each temporary pier just to carry the vertical loads during launching and even more to resist the longitudinal bearing friction forces.

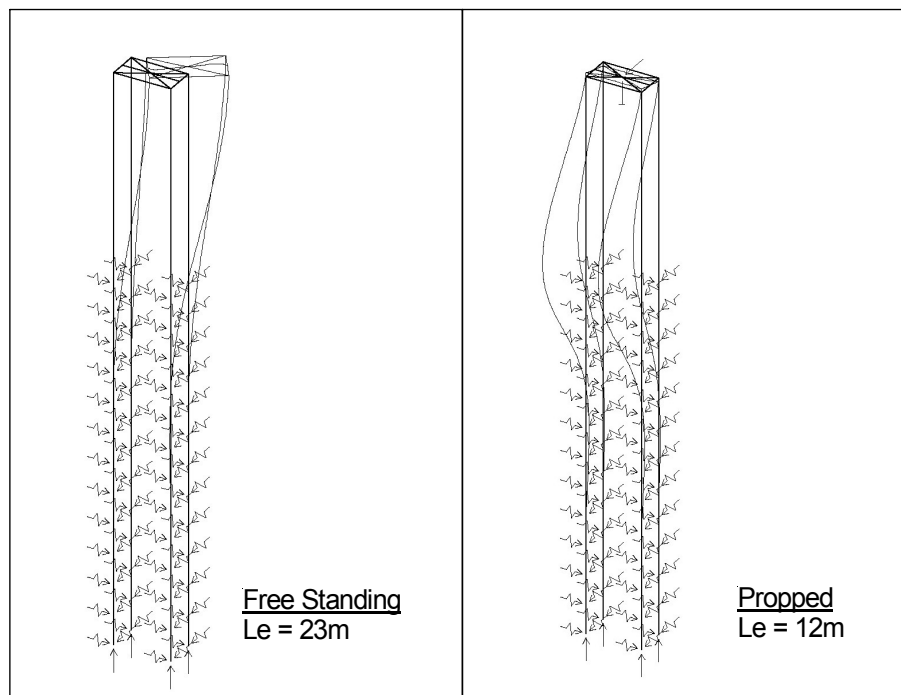


Figure 2: Temporary pier buckling modes (frame model with soil springs)

A number of pile options were considered, including the use of raking piles and bracing between the piles, but for all cases a minimum of seven piles was required. To meet the construction schedule, six piles was considered to be a realistically achievable upper limit.

A significant structural improvement could be made by propping the top of the temporary piers in the longitudinal and transverse directions thus halving the effective length of the piles ($L_e = 12\text{m}$) and doubling the effective member capacity to almost 70 percent of the section capacity (again refer Figure 2). In addition to improving the buckling behaviour, propping also meant the temporary piers were not required to resist the longitudinal bearing friction loads.

One option for propping the temporary piers that was considered was to connect them to the existing bridge. However connecting the two structures raised the concern that a potential failure of the temporary piers could occur with expansion or contraction movement of the existing bridge which would drag the attached temporary piers with it, thereby putting bending moments into the piles caused by forced displacement and second order effects.

4.2 Bracing

The final solution to the temporary pier problem was unusual, but suited this particular situation. It involved connecting the temporary piers in the transverse direction to the existing bridge using a flexible prop system capable of accommodating the longitudinal movements, and in the longitudinal direction to the permanent piers.

Longitudinal connection was made by running prestress strands between the temporary and permanent piers so that each temporary pier was tied to both adjacent permanent piers. By bracing in this way, each temporary pier was able to be constructed with just four piles. Figure 3 shows the temporary pier arrangement with cable bracing to the permanent piers and lateral prop to the existing bridge.

Although bracing dramatically improved the performance of the temporary piers, the safety of the system was still reliant on the soft soil providing some lateral restraint to the piles. A simple test procedure was developed to assess the actual insitu restraint conditions. The test, carried out at three of the critical temporary piers, involved strapping two piles together near the top of the piles and pulling them towards each other to simulate lateral loading on the individual piles. Measurements taken for increasing levels of loading and sustained loading were used to assess the immediate and long term soil restraint respectively. The results for each test showed that the short term stiffness of the soil was considerably higher than the long term stiffness due to creep effects but also proved that the actual lateral restraint of the piles in the long term was safely higher than the original design predictions. This gave the design team considerable confidence to proceed with the four pile arrangement.

To ensure each temporary pier would stand vertical with the strands tensioned to a relatively uniform stress, a sequence had to be derived for stressing the cable bracing at each temporary pier. This calculation required an estimate of the effective

stiffness of the pier, including interaction with the soil, for loading in the longitudinal direction which was then modified for the stiffness of the other connected piers and for each strand as it was added. The stiffness interaction model was used to predict the stress in the strands and the movement of the temporary piers during each stage of stressing but was sensitive to the geotechnical assessment of the lateral soil spring values.



Figure 3: Temporary pier with cable bracing to adjacent permanent piers

The movements of each temporary pier were measured during stressing of the cables which provided additional confirmation of the soil stiffness values and enabled further calibration of the temporary pier models, carried out as the stressing operation proceeded. The piers were stressed from alternating sides hence stressing for each stage pulled the temporary pier in one direction while the subsequent stage pulled it back in the opposite direction. For each temporary pier, movement from the first stage of stressing was recorded and reported back to the designer who used the information to reassess the effective pier stiffness, which was then fed back into the model to generate a revised stressing sequence. In all cases, the results again confirmed that the geotechnical engineer's estimate of the lateral soil stiffness values was safely conservative.

4.3 Presetting the Supports

With propping at the top, the temporary piers would theoretically work, but only marginally and it was critical that the soft alluvium stratum provided sufficient lateral restraint. To improve the safety of the system, it was decided to relieve load from the temporary piers during launching. This was achieved by setting the launching bearings on the temporary piers so that when fully loaded, they would sit lower than the adjacent permanent pier supports. Effectively, the deck support level at the permanent piers would be on the theoretical circular launch line, while the support level at the temporary piers would be a nominal 15mm below the theoretical launch line. Because the temporary piers, comprising four piles, were considerably less stiff

vertically than the permanent piers, comprising between 14 and 18 piles, the temporary pier supports were initially set 5mm above the line of the permanent pier supports to end up slightly below as required.

The relative displacement between the supports meant that additional bending moments were imposed on the deck, which had to be designed for. Modelling also had to include the effects of presetting the supports and the relative movement of the supports at all stages of launching allowing for the differential vertical stiffness of the permanent and temporary piers.

During launching, the settlements at all supports were monitored with periodic survey. The results were reported to the designer who used them to calibrate the design models, adjust subsequent vertical preset values, and carry out structural checks for various stages of construction. Shim plates provided a mechanism for adjusting the height of the temporary bearings up or down if required. In this way it was possible to ensure the structure was operating within safe ranges during the entire launch.

5. BRIDGE DECK

5.1 Two Part Construction Staging

With the substructure designed, the challenge then was to optimise the deck construction. Each incremental segment length was 25.4m, which allowed manageable concrete pour sizes and optimal speed of construction. The segment length, being one third of the main span length, also allowed the section to be changed to incorporate thicker webs over the permanent piers, which meant the section could be kept relatively light within the spans.

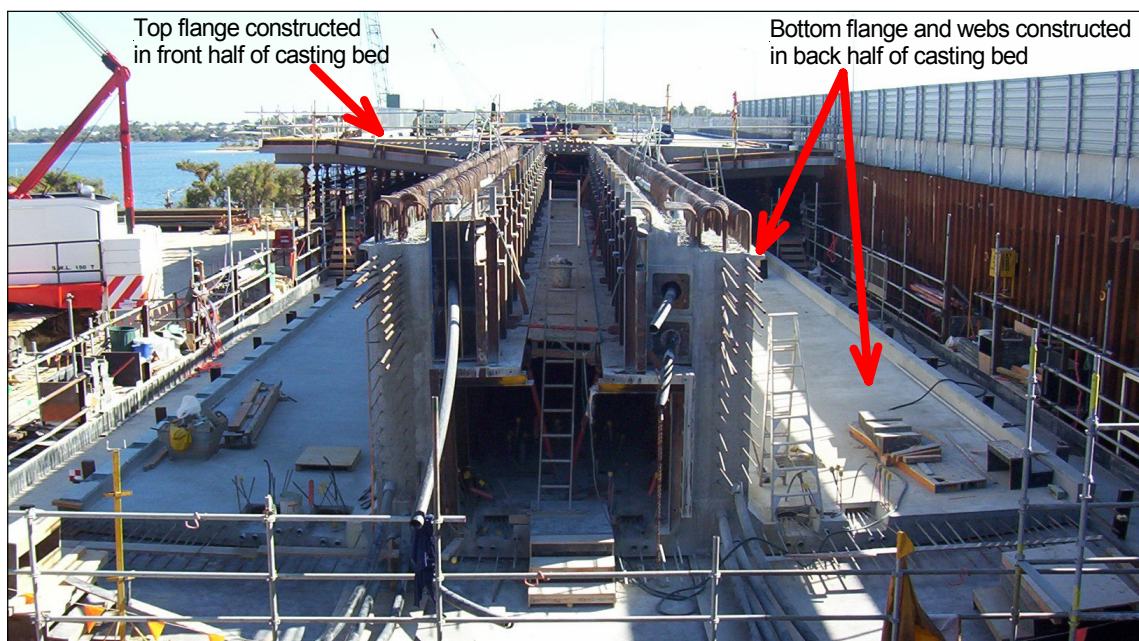


Figure 4: Staging of deck construction in casting bed

The segment length was also selected to achieve a two part construction staging of the section within the available area at the launching end. The casting bed was two segments in length to allow for simultaneous work on two fronts. The bottom flange and webs were cast in the back half of the casting bed. Once completed, the deck was launched thereby shifting the partially completed section to the front of the casting bed. While the top flange of the segment was being completed in the front half of the casting bed, the bottom flange and webs of the next segment were being constructed in the back half (refer Figure 4).

5.2 Concentric Prestress

To further improve the speed of construction, Leighton Contractors sought to minimise the launching prestress operations. This was achieved by running each concentric prestress cable for three segment lengths equal to one main span length (76m). Hence for the construction of each segment only one third of the concentric prestress had to be applied.

This meant that the completed section had just one third of the full concentric prestress when launched from the casting bed and did not have the full prestress until the two subsequent segments had been constructed and stressed. Analysis showed that the section could carry the loads from the first two launches without the full concentric prestress. Again, modelling included the presetting and subsequent settlement of the supports as the bridge was launched over. The use of temporary piers facilitated the concentric prestress arrangement by reducing the total prestress required and lowering the stresses in the deck over the length without full prestress.

6. COMPLETION OF NEW BRIDGE

With launching completed and the bridge securely anchored in position, a number of finishing works were still required to make the bridge operational. Included in these were:

- Bearing changeover at permanent piers. This operation required the bridge to be jacked off the launching bearings and lowered onto the permanent bearings.
- Application of continuity prestress. The effect of applying the continuity prestress was to lift the bridge at the temporary pier supports thus reducing the load at these supports and shifting load to the permanent piers.
- Lowering of temporary piers, transferring all load to the permanent piers.
- Deck works such as construction of barriers and asphaltting applying additional load to the structure.

Careful analysis of these works was required to ensure the bridge was not overstressed and the temporary piers were not overloaded during the process. However it was important not to place too many restrictions on the staging of the works to facilitate construction on a number of simultaneous fronts to meet the schedule.

So as not to overload the launching bearings at the permanent piers, the bearing changeover at each pier was required to be carried out prior to applying any of the continuity prestress in the spans adjacent to the pier. However, with only concentric prestress applied and with the permanent pier supports already preset higher than

the temporary piers, a limit had to be set on the jacking height to ensure the bridge capacity was not exceeded.

The continuity prestress was applied in stages after the permanent bearings in the adjacent piers had been installed and before the temporary piers could be lowered. To ensure the final deck works were not adversely affected by subsequent deck displacements, particularly the barriers, a restriction was put in place which limited deck works to at least one span away from any temporary piers still supporting the deck.

Figure 5 shows the bridge site before, during and after construction of the new Mount Henry Bridge which was opened to traffic in January 2006. Note that the temporary piers are still in place but are not supporting the bridge deck in the final photo.



Figure 5: Before, during and after construction of new Mount Henry Bridge

7. CONCLUSION

Although incremental launching is not a new technology in bridge construction, each job has its own challenges and launching the New Mount Henry Bridge was no exception. The positioning of the new bridge partially overhanging the old bridge necessitated the use of temporary piers to control deflections. Due to the slenderness of the piles in the soft river alluvium, the temporary piers required a unique bracing system which utilised the reserve strength of the permanent piers and the adjacent bridge.

Continuous monitoring during construction helped to confirm the design assumptions and improve predictions of structural behaviour for subsequent construction phases. Testing and monitoring of the insitu lateral temporary pier behaviour enabled a firmer understanding of the soil properties, and recording the vertical pier movements enabled the designer to continuously check that the piers and deck were not overloaded.

The efficient design of the temporary piers, the two part construction staging of the deck, and the concentric prestress arrangement all contributed to the swift construction of the new Mount Henry Bridge, which was opened to traffic two years after the design and construct contract was let.

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